

STRUCTURAL ANALYSIS OF THE SUPERSTRUCTURE OF THE STA 109+873 OVERPASS BRIDGE ON THE BETUNG–JAMBI TOLL ROAD PROJECT, SECTION 2B

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Abstract

This study evaluates the superstructure of the overpass bridge at STA 109+873 on the Betung-Jambi Toll Road Section 2B. The analysis focuses on the reinforced-concrete deck slab, support and intermediate diaphragms, and a 35.80 m prestressed PC I-girder H-170 system. Secondary project data and shop drawings were assessed using RSNI T-12-2004, SNI 1725:2016, SNI 2833:2016 as a general bridge reference, and Bridge Technical Planning Guideline No. 02/M/BM/2021. Structural checks covered loading, reinforcement, prestress requirements and losses, concrete stress, flexural capacity, shear capacity, and deflection. The 250 mm deck slab requires D16-125 flexural reinforcement and D13-200 distribution reinforcement; its factored shear and deflection remain below the permitted limits. The 400 x 650 mm support diaphragm and 250 x 1250 mm intermediate diaphragm satisfy strength requirements with 6D19 main reinforcement. The PC I-girder uses four tendons with 76 strands. Its calculated flexural resistance, shear resistance, stress levels, and service deflection satisfy the adopted design criteria. The results indicate that the analyzed superstructure meets the required strength, stiffness, and serviceability provisions for the investigated loading scope.

Keywords: Bridge superstructure; Deck slab; Overpass; PC I-girder; Prestressed concrete.

INTRODUCTION

Bridges are critical elements of road infrastructure because they maintain continuity across natural or constructed obstacles while transferring traffic and environmental actions safely to their supports. In a toll-road network, the structural performance of overpasses is particularly important because the bridge must accommodate one traffic stream without interrupting the principal roadway below. The superstructure receives vehicle loads directly and therefore governs several practical design decisions, including deck thickness, transverse load distribution, girder spacing, prestressing demand, and serviceability control. A bridge superstructure must consequently be checked not only for ultimate strength but also for stress limits, deformation, cracking-related behavior, and the effects of time-dependent prestress losses (Struyk & Veen, 1984; Siswanto, 1999).

The Betung-Jambi Toll Road forms part of the Trans-Sumatra road development program. Within Section 2B, an overpass is located at STA 109+873 near Simpang Bayat, Bayung Lencir District, Musi Banyuasin Regency, South Sumatra. The complete bridge is 75 m long and consists of two spans supported by two abutments and one pier. This study evaluates one representative 35.80 m span because the adjacent span is geometrically symmetrical. The analyzed superstructure comprises a reinforced-concrete deck slab, transverse diaphragms, and three prestressed PC I-girders with a nominal depth of 1.70 m and a center-to-center spacing of 2.30 m. The bridge width is 7.50 m, including two 3.25 m traffic lanes and edge parapet zones, and the bridge does not include a pedestrian sidewalk.

Prestressed I-girders are widely used for medium bridge spans because prestressing reduces service-level tensile stress and permits relatively slender sections. Their structural efficiency, however, depends on a coordinated design of the girder section, tendon profile, strand quantity, composite deck action, and reinforcement for flexure and shear. In post-tensioned or segmental applications, friction, anchorage seating, elastic shortening, creep, shrinkage, and strand relaxation reduce the initial prestressing force. These losses directly influence concrete stresses and deflection,

so they should be considered together with load combinations rather than treated as an isolated calculation (Aboe, 2006; AASHTO, 2017).

Indonesian bridge design practice combines strength and serviceability requirements from national standards and technical guidance. SNI 1725:2016 provides bridge loading rules, including self-weight, additional dead load, traffic load, wind, braking, temperature, and relevant load combinations. RSNI T-12-2004 provides concrete bridge design provisions, while the Practical Bridge Technical Planning Guideline No. 02/M/BM/2021 provides procedures used in this study for prestressed members, reinforcement, prestress losses, flexural resistance, shear resistance, and deflection. SNI 2833:2016 was listed among the project references; however, the numerical scope of this study explicitly excludes seismic-load analysis and focuses on the superstructure under the non-seismic actions specified in this study.

Previous Indonesian studies have examined related bridge systems but under different structural configurations. Simamora et al. (2022) analyzed an underpass with both superstructure and substructure components; Saputra et al. (2020) assessed an existing bridge using the rating-factor method; Nusantara et al. (2022) redesigned a shorter bridge using PC I-girders; and Nasution (2023) compared prestressed I-girder and box-girder alternatives. Other studies examined precast I-girder evaluation, external prestressing, and project-specific redesigns with different span lengths, girder spacing, material strengths, and tendon arrangements (Azhara & Nuranita, 2023; Panjaitan, 2024; Suryawinata & Widyaningsih, 2025). These differences limit direct transfer of their numerical results to the STA 109+873 bridge.

The present study therefore addresses a project-specific question: whether the selected dimensions, reinforcement, and prestressing arrangement of the STA 109+873 overpass superstructure satisfy the adopted Indonesian design criteria for the investigated loading scope. The analysis is organized around three connected components. First, the deck slab is evaluated for minimum thickness, factored moments, reinforcement, punching shear, and deflection. Second, the support and intermediate diaphragms are evaluated as transverse simple-beam elements for self-weight effects, reinforcement ratios, flexure, and shear. Third, the PC I-girder is evaluated for section properties, composite action, load effects, tendon requirements, prestress losses, concrete stresses, flexural and shear resistance, and deflection.

The contribution of the paper is not a new bridge-design theory; rather, it is an integrated verification of a real toll-road overpass configuration using a consistent set of project data and design rules. By condensing the calculations into a journal format, the paper identifies the controlling structural checks and presents the numerical margins that support the final design assessment. The results are intended to be useful for comparable PC I-girder applications and for students or practitioners who need a transparent example of how deck, diaphragm, and prestressed-girder checks interact within a single superstructure design.

Table 1. Principal standards and analytical scope

Reference	Primary use in this study	Scope note
RSNI T-12-2004	Concrete bridge design and reinforcement checks	Used for concrete member design criteria.
SNI 1725:2016	Permanent, traffic, wind, braking, temperature, and load combinations	Main loading standard for the superstructure.
SNI 2833:2016	General seismic bridge reference	Cited in this study, but seismic demand was excluded from the numerical scope.
No. 02/M/BM/2021	Bridge technical planning, prestressed-girder checks, losses, flexure, shear, and deflection	Primary practical guidance for detailed member verification.
AASHTO LRFD 2017	Selected prestress-loss parameters, including low-relaxation strand treatment	Used where cited in this study for prestressing calculations.

LITERATURE REVIEW

Bridge Superstructure and Prestressed Concrete

A bridge is commonly divided into a superstructure and a substructure. The superstructure carries traffic and other direct actions, whereas the substructure transfers the resulting reactions to the foundation and supporting ground. For the overpass studied here, the relevant superstructure components are the deck slab, diaphragms, and longitudinal girders. The deck distributes wheel and surface loads to the main girders. Diaphragms provide transverse

restraint and help maintain the relative position of adjacent girders. The longitudinal girders carry the dominant spanwise bending and shear effects and, after deck hardening, work with the slab as a composite section.

Prestressed concrete is used to introduce compressive stress into a member before or during service so that tensile stress caused by external loading is reduced. In post-tensioning, ducts are cast into the concrete and the tendons are stressed after the concrete reaches the required strength. The stressed tendons are anchored at the member ends and the ducts can be grouted to protect the steel and develop bond. This study uses uncoated seven-wire low-relaxation strand for the PC I-girder, which is consistent with the conventional strand form described in prestressed-concrete references (Aboe, 2006).

The structural advantage of a PC I-girder comes from placing material efficiently relative to the neutral axis. The flanges provide compression and tension zones, while the relatively thin web transfers shear. Prestressing further improves span efficiency by counteracting part of the gravity-load bending. The resulting design still requires conventional reinforcement, especially for web shear, local reinforcement, and minimum longitudinal steel. Therefore, a prestressed-girder design cannot be judged only by the calculated tendon force; its adequacy depends on combined checks of concrete stress, flexural capacity, shear capacity, and service deflection.

Bridge loading under SNI 1725:2016 is treated through limit-state combinations. Permanent actions include member self-weight and additional dead loads such as asphalt surfacing and water accumulation considered in this study. Traffic actions include lane load and truck load, together with dynamic-load amplification where applicable. Environmental actions considered in the analyzed scope include wind and temperature. Because different actions can govern different components, the study evaluates deck slab and girder responses under the combinations relevant to their design rather than relying on a single global load case (Badan Standardisasi Nasional, 2016).

Prestress Losses and Serviceability

Prestress loss is a central issue in post-tensioned bridge girders. Immediate losses arise from tendon friction, anchorage seating, and elastic shortening of the concrete. Time-dependent losses develop through concrete shrinkage and creep and through relaxation of the prestressing steel. This study organizes the calculation into transfer, transfer-to-deck-casting, and deck-casting-to-final stages. This staged approach is useful because the member stiffness, applied loads, and composite action change during construction. The final effective prestress is then used to evaluate concrete stress and deflection at service.

The practical significance of prestress loss is visible in the serviceability checks. Excessive loss reduces the beneficial compression and upward camber generated by prestressing, potentially increasing tensile stress and downward deflection under traffic. Conversely, very high initial prestress can cause excessive compression or upward deformation during transfer. A satisfactory design therefore requires both early-age and final-age checks. This study performs stress checks at transfer, during construction, and in the service condition and also evaluates deflection before and after composite action.

The adopted bridge-planning guideline provides indicative prestress-loss ranges for several structural cases, while the detailed calculation in this study develops individual loss components. The calculated total loss is later compared with the resulting effective prestress rather than being treated merely as a percentage target. This distinction is important because bridge safety depends on the stresses and capacities after losses, not on a loss percentage in isolation.

Evidence from Previous Studies

Recent Indonesian bridge studies illustrate the sensitivity of prestressed-girder results to span and configuration. Nusantara et al. (2022) redesigned a 16 m bridge using a PC I-girder and reported a total prestress loss of 28.10% together with a service deflection below its allowable limit. Panjaitan (2024) analyzed a 40.80 m PC I-girder H-170 bridge and used five tendons with 92 strands; the reported prestress loss, flexural resistance, shear resistance, and deflection all met the criteria adopted in that study. Purnama (2025) reported a 250 mm deck slab, D16-150 main reinforcement, D13-200 distribution reinforcement, and five tendons with 92 strands in a separate bridge redesign. These values provide context, but they are not transferable directly because the geometric, material, and load parameters differ.

Azhara and Nuranita (2023) evaluated a precast concrete I-girder on the Cisumdawu toll road using SNI 1725:2016 and RSNI T-12-2004, reporting acceptable stress and deflection behavior for the investigated structure. Nasution (2023) compared PC I-girder and prestressed box-girder behavior and found differences in moment, shear, tendon demand, and prestress loss. Suryawinata and Widyaningsih (2025) compared externally prestressed reinforced-concrete girders with precast I-girders and likewise showed that girder type changes weight, capacity

ratios, and deformation. Collectively, these studies reinforce the need for project-specific analysis rather than design-by-analogy.

Table 2. Selected previous studies and their relation to the present analysis

Study	Bridge/system	Main reported focus	Relation to STA 109+873
Simamora et al. (2022)	40 m underpass bridge	Superstructure and substructure analysis	Broader structural scope and different girder geometry.
Saputra et al. (2020)	Existing prestressed bridge	Rating-factor capacity assessment	Evaluation of existing capacity rather than new superstructure design.
Nusantoro et al. (2022)	16 m PC I-girder	Tendon layout, loss, deflection, reinforcement	Shorter span; useful comparison for prestress-loss and serviceability concepts.
Azhara & Nuranita (2023)	Precast concrete I-girder	Stress ratio and deflection evaluation	Comparable girder concept under SNI-based checks.
Nasution (2023)	PC I-girder vs. box girder	Moment, shear, tendon demand, prestress loss	Highlights section-type effects on structural response.
Panjaitan (2024)	40.80 m PC I-girder H-170	Deck, diaphragm, tendons, loss, flexure, shear, deflection	Closest section depth, but different span, width, spacing, and material parameters.
Suryawinata & Widyaningsih (2025)	Externally prestressed RC vs. precast I-girder	Capacity ratio, deflection, girder weight	Shows sensitivity of response to structural system.

The gap addressed by the present study is therefore specific and practical. The STA 109+873 bridge uses a 35.80 m span, a 1.70 m-deep PC I-girder, a relatively wide 2.30 m girder spacing, and a deck-diaphragm arrangement defined by project shop drawings. The article integrates the checks for these components using the same set of project loads and material data, rather than focusing on only one structural element or comparing alternative girder types. This integrated framing is important because a superstructure can satisfy a girder-level strength check while still being governed by deck-slab reinforcement, diaphragm detailing, prestress loss, concrete stress, or service deflection. The research questions are therefore component-specific but evaluated as part of one load path: Do the deck thickness and reinforcement satisfy strength and deflection requirements? Do the diaphragm dimensions and reinforcement provide adequate transverse strength? And does the selected PC I-girder, including its tendon system and nonprestressed reinforcement, remain within allowable stress, flexural, shear, and deflection limits?

METHOD

Research Design and Data Source

The study uses an analytical design-verification approach based on secondary project data. The primary source is the shop-drawing and technical information for the Overpass STA 109+873 on the Betung-Jambi Toll Road Section 2B, supplied through the project environment described in this study. The analysis does not include field load testing, material sampling, or monitoring. Instead, project geometry and material specifications are translated into structural parameters, loads are determined according to the adopted standards, and the resulting demands are compared with calculated capacities or allowable service limits.

The project location is near Simpang Bayat, Bayung Lencir District, Musi Banyuasin Regency, South Sumatra. The complete overpass is 75 m long with two spans, but the analysis uses one 35.80 m span because the second span is symmetrical. Only the superstructure is included. The substructure, foundation, construction-cost estimate, construction schedule, and seismic-load calculation are outside the numerical scope. This boundary allows the paper to concentrate on the behavior of the deck slab, diaphragms, and prestressed longitudinal girders under the specified permanent, traffic, wind, braking, and temperature effects.

The analytical sequence follows the workflow used in this study: literature and standard review, collection of project data, definition of design parameters, load calculation, structural demand calculation, reinforcement and prestress design, structural verification, and conclusion. The deck slab is treated as a transverse slab strip for design, the diaphragms are treated as simple beams for their self-weight and reinforcement checks, and the PC I-girder is evaluated first as a noncomposite prestressed member and subsequently as a composite girder with the cast deck.

Table 3. Principal bridge geometry and material parameters

Parameter	Value	Unit / note
Analyzed span length	35.80	m
Total bridge width	7.50	m
Traffic-lane width	3.25	m per lane
Deck slab thickness	250	mm
Girder type	PC I-girder H-170	1.70 m nominal depth
Girder spacing	2.30	m
Number of longitudinal girders	3	units
Deck/diaphragm concrete strength, f'_c	30	MPa
Prestressed-girder concrete strength, f'_{cg}	50	MPa at 28 days
Girder concrete strength at transfer, f'_{cig}	40	MPa
Prestressing strand	12.7 mm seven-wire low-relaxation	$f_{pu} = 1860$ MPa; $f_{py} = 1674$ MPa
Deck/diaphragm reinforcing steel yield strength	520	MPa
Girder nonprestressed reinforcing steel yield strength	400	MPa

Load and Member Analysis

For the deck slab, this study considers self-weight, asphalt and rainwater as additional dead loads, truck loading, wind acting on vehicles, and temperature effects. The 250 mm slab gives a self-weight of 6.00 kN/m for the one-metre design strip, while the asphalt and rainwater contributions total 1.59 kN/m. The design wheel load after a 40% dynamic amplification is 157.50 kN. A wind-related point effect of 1.502 kN and a temperature difference of 12.5 degrees Celsius are included in the deck analysis. Moment diagrams were generated for the individual load effects, after which the applicable SNI load combinations were formed.

For the diaphragms, the support element is 400 x 650 mm with a 1.60 m design length, while the intermediate diaphragm is 250 x 1250 mm with a 2.10 m design length. Both use 30 MPa concrete and 520 MPa reinforcing steel. The diaphragms are idealized as simple beams and are checked for flexural reinforcement, distribution reinforcement, and shear reinforcement. The support and intermediate self-weight line loads after the applied ultimate factor are 8.112 kN/m and 9.75 kN/m, respectively.

For the longitudinal girder, section properties are evaluated for the noncomposite PC I-girder and for the composite girder with the deck slab. The prestressed concrete has a 28-day modulus of approximately 38,006.989 MPa and a transfer modulus of 33,994.485 MPa. The girder cross-sectional area is 0.672 m² and its noncomposite moment of inertia is 0.238 m⁴. Prestressing is provided by 12.7 mm strands arranged in tendons. The tendon requirement is determined from the prestress demand and strand capacity, followed by tendon-profile, jacking-force, and loss calculations.

Prestress losses are evaluated in stages. Immediate losses include friction, anchorage seating, and elastic shortening. Long-term components include girder shrinkage, strand relaxation, girder creep, and the deck-related shrinkage effect used in this study. The resulting effective prestress is then used in concrete stress checks at transfer, construction, and service. Strength verification compares reduced flexural and shear resistance with the governing ultimate demand, and serviceability verification compares calculated deflections with the limits used in this study.

All reported numerical results in this paper are derived from the submitted source material. The article reorganizes and translates the calculations into a journal format but does not introduce new project measurements or substitute alternative design standards. Where this study reports both preliminary assumptions and later detailed calculations, the detailed result is presented as the final analytical value and the earlier assumption is described only as a preliminary design estimate.

Table 4. Analytical sequence and acceptance checks

Component	Demand calculations	Primary verification
Deck slab	Self-weight, additional dead load, truck, wind, temperature; factored support and span moments	Minimum thickness; flexural and distribution reinforcement; punching shear; deflection
Support/intermediate diaphragms	Factored self-weight; simple-beam moment and shear	Reinforcement ratio; flexure; minimum shear reinforcement and spacing
PC I-girder	Noncomposite and composite properties; permanent, traffic, wind, braking, and temperature effects	Tendon demand; prestress loss; transfer/construction/service stress; flexure; shear; deflection

Verification Criteria and Analytical Consistency

The verification framework treats each superstructure component according to its governing structural response rather than applying a single global acceptance measure. For the deck slab, the adopted thickness is checked against the minimum thickness provisions used in this study, after which the governing positive and negative moments are used to determine flexural reinforcement. Distribution reinforcement is then evaluated as secondary reinforcement in the longitudinal direction. The slab verification is completed by a punching-shear comparison and a serviceability deflection check. This sequence follows the design logic of RSNI T-12-2004 and the practical bridge-design guidance used in the original calculations (Badan Standardisasi Nasional, 2004; Kementerian Pekerjaan Umum dan Perumahan Rakyat, 2021).

For the diaphragms, the analytical idealization is deliberately limited to a simply supported beam subjected to its own weight, consistent with this study. The calculated flexural demand is compared with the reinforcement provided in the support and intermediate diaphragms, while shear reinforcement is checked against the required and maximum spacing criteria. Because the diaphragm calculation is a local member check, the paper does not reinterpret the diaphragm as a global transverse distribution model. This distinction is important for maintaining consistency with the submitted source and avoiding claims that are not supported by the original analysis.

The girder verification is organized around the change from noncomposite to composite behavior. Before the deck becomes composite, the PC I-girder carries its self-weight and construction-related actions with the initial prestressing force. After composite action is established, the transformed section properties are used for the additional dead load and traffic-related effects. The effective deck width, section modulus, and moments of inertia are therefore treated as stage-dependent quantities. This staged approach is necessary because the stress state at transfer cannot be assessed with the same section properties and prestressing level used for the final service condition.

Prestressing is checked through tendon demand, jacking force, and detailed loss calculations. The tendon count is rounded upward to a constructible arrangement, and the reported jacking level is checked against the 80% limit used in this study. Immediate losses comprise friction, anchorage seating, and elastic shortening. Time-dependent losses comprise shrinkage, creep, and relaxation over the transfer-to-deck and deck-to-final intervals, including the deck-shrinkage effect accounted for in the source calculation. The resulting effective prestress is then used in the subsequent stress, flexural, shear, and deflection checks rather than relying solely on the preliminary assumed loss percentage.

Concrete stresses are examined at transfer, construction, and final service stages. This study compares compressive and tensile stresses with the stage-specific limits adopted from its design references. Ultimate flexural verification requires the reduced flexural resistance to exceed the governing factored moment and also checks the minimum flexural-resistance criterion. Ultimate shear verification compares the factored shear demand with the reduced nominal resistance, including concrete, prestressing, and transverse-reinforcement contributions. Serviceability is assessed independently through transfer camber, traffic-related deflection, and the final Service I deflection. This separation of strength and serviceability checks prevents a satisfactory ultimate capacity from being interpreted as automatic compliance with deformation or stress criteria.

The calculation record also provides an internal consistency check for the article. Loads, material properties, section dimensions, reinforcement, tendon configuration, prestress losses, stresses, and member capacities are reported using the same units and component definitions as those used in this study. Values are not recalculated with alternative standards or revised assumptions. Where the original study excludes seismic loading, the present article preserves that boundary and reports the work as a non-seismic superstructure verification. Consequently, the findings should be interpreted as evidence of compliance within the analyzed loading scope, not as a substitute for a complete

bridge-design certification that would also require substructure, foundation, seismic, construction-stage, and project-specific evaluations.

RESULTS AND DISCUSSION

Deck Slab Performance

The minimum deck thickness check gave a required value below 200 mm, while the adopted thickness is 250 mm. The 250 mm slab therefore satisfies the minimum thickness requirement used in this study and provides an effective depth of approximately 202 mm after allowing for cover and reinforcement. The load analysis produced individual moments for self-weight, additional dead load, truck load, wind, and temperature, which were then combined under the SNI 1725:2016 limit states. The governing strength combination was Strength I.

Under the governing combination, the reported ultimate moment is 126.627 kNm at the support region and 106.848 kNm in the span region. These demands lead to D16 reinforcement at 125 mm spacing in both regions. The distribution reinforcement is D13 at 200 mm spacing. Using the same reinforcement in the support and span regions simplifies detailing while still satisfying the calculated steel requirement. The result is somewhat more closely spaced than the D16-150 deck reinforcement reported in several comparison studies, which is consistent with the relatively large 2.30 m girder spacing and the specific load distribution of the STA 109+873 bridge (Purnama, 2025; Panjaitan, 2024).

The deck punching-shear check gives a factored shear demand of 283.500 kN. The nominal slab shear resistance is 1,014.922 kN and the reduced design resistance used in this study is 761.191 kN. Thus, the factored demand is well below the design resistance. The deflection check is also satisfactory: the calculated total deck deflection is 1.958 mm compared with an allowable value of 9.583 mm. These two checks indicate that the selected slab thickness is not controlled by punching shear or service deflection under the analyzed loading.

From a design perspective, the slab result shows the value of checking strength and serviceability together. A 250 mm slab is thicker than the calculated minimum, but the final reinforcement demand is still governed by the factored bending moments. The low utilization in punching shear and deflection does not imply that reinforcement can be reduced without recalculation, because the flexural demand and minimum reinforcement provisions remain independent requirements.

Table 5. Deck slab design and verification results

Item	Demand / adopted value	Capacity / limit	Assessment
Slab thickness	250 mm	Minimum required thickness = 200 mm	Meets thickness requirement
Ultimate support moment	126.627 kNm	Designed with D16-125	Adequate reinforcement
Ultimate span moment	106.848 kNm	Designed with D16-125	Adequate reinforcement
Distribution reinforcement	D13-200	Required distribution steel	Provided
Punching shear	$V_u = 283.500$ kN	$V_r = 761.191$ kN	$V_u < V_r$
Total slab deflection	1.958 mm	9.583 mm allowable	Within limit

Diaphragm Design

The diaphragm analysis distinguishes the support diaphragm from the intermediate diaphragm. The support diaphragm has a 400 mm thickness and a 650 mm depth, while the intermediate diaphragm is 250 mm thick and 1,250 mm deep. The corresponding factored self-weight line loads are 8.112 kN/m and 9.75 kN/m. Treated as simple beams, these loads produce local design moments of 2.596 kNm and 5.375 kNm, respectively. The larger depth of the intermediate diaphragm provides a greater lever arm for the transverse element even though its width is smaller.

For the support diaphragm, the required flexural steel area is 621.923 mm², and the selected detail is 6D19 with a provided area of approximately 1,701.172 mm². The calculated reinforcement ratio remains between the lower and upper limits used in this study. Distribution reinforcement is 2D13. The factored shear demand is 6.490 kN, which is far below the concrete shear check; D13-200 shear reinforcement is nevertheless provided as the adopted detail and satisfies the minimum and spacing requirements. For the intermediate diaphragm, the calculated flexural steel requirement is 792.548 mm² and the same 6D19 main reinforcement is adopted. Distribution reinforcement is 4D13, and D13-200 is used for shear reinforcement. The factored shear demand of 10.238 kN remains below the

corresponding concrete shear threshold. The two diaphragm types therefore meet the reinforcement-ratio, flexural, and shear criteria used in the analysis. The diaphragm checks also demonstrate an important distinction between local member design and global girder loading. The self-weight of the diaphragms contributes to longitudinal girder demand, while the diaphragm reinforcement itself is designed from the local transverse idealization. Keeping these two roles separate prevents the equivalent distributed diaphragm weight used in the girder analysis from being confused with the local simple-beam moment used for diaphragm reinforcement.

Table 6. Diaphragm dimensions, actions, and reinforcement

Parameter	Support diaphragm	Intermediate diaphragm
Section size	400 x 650 mm	250 x 1250 mm
Design length	1.60 m	2.10 m
Factored self-weight line load	8.112 kN/m	9.75 kN/m
Local simple-beam moment	2.596 kNm	5.375 kNm
Required flexural steel	621.923 mm ²	792.548 mm ²
Main reinforcement	6D19	6D19
Distribution reinforcement	2D13	4D13
Shear reinforcement	D13-200	D13-200
Factored shear demand	6.490 kN	10.238 kN

Prestressed Girder and Tendon Requirement

The main longitudinal system uses three PC I-girders, each 1.70 m deep and spaced at 2.30 m. The girder concrete has a specified 28-day strength of 50 MPa and a transfer strength of 40 MPa. The noncomposite cross-sectional area is 0.672 m² and the moment of inertia is 0.238 m⁴. After the deck hardens, the slab participates in composite action and changes the section centroid and stiffness used in service stress and deflection calculations. The tendon design uses uncoated seven-wire low-relaxation strand with a nominal diameter of 12.7 mm, ultimate tensile strength of 1,860 MPa, and yield strength of 1,674 MPa. The project-specific tendon arrangement contains 19 strands per tendon. The calculated tendon requirement is 3.972 tendons, rounded up to four tendons. This results in 76 strands and a total nominal strand area of 7,501.2 mm². The calculated jacking ratio is 79.44%, remaining below the 80% limit used in this study, and the corresponding jacking force is 11,090.812 kN.

The four-tendon arrangement differs from some of the comparison studies. Panjaitan (2024) and Purnama (2025), for example, reported five tendons and 92 strands for their respective bridge designs. The present bridge uses fewer strands because the demand, material strengths, geometry, and tendon specification are different. This comparison underscores why tendon number should follow the actual stress demand and strand capacity rather than a fixed rule based only on girder depth. Before the detailed loss calculation, this study used a 26% loss assumption as a preliminary estimate based on the practical guideline. The subsequent staged analysis yielded a larger calculated loss, so the final stress and capacity checks are more informative than the preliminary percentage. The detailed loss components are discussed below.

Table 7. Prestressing system and calculated tendon requirement

Parameter	Reported value
Girder concrete strength at 28 days	50 MPa
Girder concrete strength at transfer	40 MPa
Strand type	Uncoated seven-wire low-relaxation
Strand diameter	12.7 mm
Strand ultimate strength, fpu	1860 MPa
Strand yield strength, fpy	1674 MPa
Strands per tendon	19
Required tendons	3.972 -> 4 tendons
Total strands	76
Total nominal strand area	7501.2 mm ²
Jacking ratio	79.44%
Jacking force	11090.812 kN

Prestress Loss and Effective Prestress

The detailed prestress-loss calculation is one of the most substantial parts of this study. At transfer, friction contributes 46.995 MPa, anchorage seating contributes 47.880 MPa, and elastic shortening contributes 47.413 MPa. The combined immediate loss is therefore 142.288 MPa. Between transfer and deck casting, the reported components are 156.672 MPa from girder shrinkage, 8 MPa from strand relaxation, and 330.468 MPa from girder creep. For the later stage from deck casting to final time, the reported values are 55.527 MPa for girder shrinkage, 8 MPa for relaxation, -4.181 MPa for the stated creep component, and a 49.031 MPa prestress increment associated with deck shrinkage in the sign convention used in this study.

After combining the staged terms, the long-term loss is 505.455 MPa and the total calculated prestress loss is 647.743 MPa, equivalent to 34.825%. This value is higher than the preliminary 26% estimate adopted earlier in the tendon-sizing stage. It is also higher than the prestress-loss percentages reported in several comparison studies, such as 28.10% by Nusantara et al. (2022) and approximately 25.97-27.40% in the two girder alternatives reported by Nasution (2023). Direct numerical comparison should be made cautiously because the studies use different geometries, tendon profiles, material properties, and time-dependent assumptions.

Using the detailed loss calculation, the reported effective prestressing stress at the evaluated section is 315.187 MPa, below the stated maximum effective-stress criterion of 1,339 MPa. With the total strand area, the corresponding final effective force is approximately 2,364.279 kN. The loss calculation therefore does not end the assessment; the next step is to confirm that the remaining prestress produces acceptable concrete stresses and sufficient flexural and shear resistance. The difference between the preliminary loss assumption and the detailed result is analytically important. It demonstrates that preliminary percentage-based sizing should be followed by a staged loss calculation before final serviceability conclusions are drawn. In this study, the larger calculated loss did not invalidate the girder because the subsequent transfer, construction, service stress, flexural, shear, and deflection checks remained within the limits applied in this study.

Table 8. Prestress-loss components at midspan

Stage	Loss component	Value (MPa)
Transfer	Friction, Δf_{pF}	46.995
Transfer	Anchorage seating, Δf_{pA}	47.880
Transfer	Elastic shortening, Δf_{pES}	47.413
Transfer to deck casting	Girder shrinkage, Δf_{pSR}	156.672
Transfer to deck casting	Strand relaxation, Δf_{pR1}	8.000
Transfer to deck casting	Girder creep, Δf_{pCR}	330.468
Deck casting to final	Girder shrinkage, Δf_{pSD}	55.527
Deck casting to final	Strand relaxation, Δf_{pR2}	8.000
Deck casting to final	Girder creep, Δf_{pCD}	-4.181
Deck casting to final	Prestress increment from deck shrinkage, Δf_{pSS}	49.031 credited
Total	Calculated total loss	647.743 MPa = 34.825%

Concrete Stress Verification

This study establishes separate allowable stresses for transfer and service. At transfer, the stated compressive limit is 24 MPa in magnitude and the tensile limit is 1.581 MPa. The calculated top-fiber stress is -3.337 MPa and the bottom-fiber stress is -23.896 MPa. Under the sign convention used in the calculations, both are compressive and remain within the 24 MPa limit, although the bottom-fiber value is close to the permitted compression and therefore represents an important early-age check.

During construction, after including the relevant dead-load components, the calculated top and bottom girder stresses are -14.830 MPa and -13.185 MPa, respectively. Both remain within the transfer/compressive criterion used for the construction stage. At service, the girder top-fiber stress is -11.810 MPa, below the stated 22.5 MPa compression limit. The bottom-fiber stress is +3.310 MPa in tension, which remains below the stated service tensile limit of 3.536 MPa. The composite deck stresses are -7.662 MPa and -11.965 MPa, both within the 13.5 MPa compression limit used for the slab at service.

These stress results show why the prestress-loss calculation must be coupled to staged analysis. At transfer, the critical issue is high compression caused by a relatively large prestressing force acting with limited gravity load. At service, the reduced effective prestress must still prevent excessive tensile stress at the bottom of the girder. The STA 109+873 design satisfies both conditions in the calculations.

Flexural Capacity, Shear Capacity, and Deflection

The flexural capacity check is performed at the section of maximum positive moment near midspan. Using a total strand area of 7,501.2 mm², a calculated average prestressing-steel stress of 1,813.408 MPa, and a strength-reduction factor of 0.90, the nominal moment is 23,946.541 kNm and the reduced flexural resistance is 21,551.886 kNm. The governing ultimate moment demand is 15,865.092 kNm. The resistance therefore exceeds the demand. This study also checks the minimum flexural resistance requirement and reports a controlling minimum value of 19,365.422 kNm, which remains below the available resistance.

The shear check is made near the support, where the design shear is highest. The concrete contribution is 876.596 kN, the prestressing contribution is 38.313 kN, and the adopted D13-150 two-leg shear reinforcement provides a calculated steel contribution of 1,241.501 kN. The resulting nominal shear resistance is 2,156.410 kN, and the reduced design resistance is 1,617.308 kN. This exceeds the critical ultimate shear demand of 1,270.771 kN. The calculated concrete shear stress of 5.322 MPa is below the 6.25 MPa threshold used to determine the maximum spacing rule, and the provided 150 mm spacing is far below the calculated maximum of 1,244.673 mm.

Serviceability is checked at several stages. Before composite action, the prestressing force creates upward camber; the reported transfer-stage deflection is approximately 0.089 m upward, within the 0.119 m limit used in this study. After short-term prestress loss, the upward deflection reduces to approximately 0.0454 m. In the final composite service check, the combination of self-weight, additional dead load, prestress, traffic, braking, and wind gives a net downward deflection of 0.015 m, below the 0.119 m service limit. The lane-load-related downward

deflection is 0.0092 m, below the 0.0448 m limit used for that check. Together, the strength and deflection results indicate that the girder has reserve capacity under the analyzed load combinations without violating the serviceability limits adopted in this study. The flexural capacity margin is meaningful because the reduced resistance remains above both the ultimate demand and the minimum-resistance criterion. The shear check similarly shows a positive margin after applying the resistance factor. The service deflection is much smaller than the general service limit, while the staged camber calculations confirm that prestress-induced upward deformation also remains controlled.

Table 9. Summary of PC I-girder verification

Check	Calculated demand / response	Capacity / allowable value	Result
Transfer top stress	-3.337 MPa	24 MPa compression limit (magnitude)	Within limit
Transfer bottom stress	-23.896 MPa	24 MPa compression limit (magnitude)	Within limit
Construction top / bottom stress	-14.830 / -13.185 MPa	24 MPa compression limit (magnitude)	Within limit
Service top stress	-11.810 MPa	22.5 MPa compression limit (magnitude)	Within limit
Service bottom stress	+3.310 MPa	3.536 MPa tensile limit	Within limit
Reduced flexural resistance	$M_r = 21551.886 \text{ kNm}$	$M_u = 15865.092 \text{ kNm}$	$M_r > M_u$
Reduced shear resistance	$\phi V_n = 1617.308 \text{ kN}$	$V_u = 1270.771 \text{ kN}$	$\phi V_n > V_u$
Lane-load deflection	0.0092 m	0.0448 m	Within limit
Final Service I deflection	0.015 m	0.119 m	Within limit

Integrated Discussion and Design Implications

The component-level results are mutually consistent. The deck slab is governed primarily by flexural reinforcement rather than punching shear or deflection. The diaphragms require conventional reinforcement even though their local shear demands are relatively small. The PC I-girder is governed by a combination of prestress design, staged stress control, and ultimate flexural and shear checks. No single calculation is sufficient to justify the superstructure; adequacy follows from the complete sequence of checks.

The most notable numerical feature is the 34.825% calculated prestress loss. Because this value exceeds the preliminary 26% assumption, it would be inappropriate to assess the girder using the preliminary effective prestress alone. This study addresses this by continuing to final stress, flexural, shear, and deflection checks using the detailed prestress calculations. The service bottom-fiber tensile stress of 3.310 MPa is below the stated 3.536 MPa tensile limit, indicating that the remaining prestress is still sufficient for the service stress criterion adopted in the study.

Comparison with previous work also suggests that tendon count or loss percentage should not be treated as a universal benchmark. The four-tendon, 76-strand arrangement in this bridge differs from the five-tendon, 92-strand arrangements reported in some other 1.70 m-deep I-girder designs. The present arrangement is supported by its own demand-capacity checks. Likewise, the higher calculated loss percentage does not by itself imply structural inadequacy because the governing decision variables are the resulting stresses, capacities, and deflections. Differences in span, girder spacing, tendon geometry, concrete age, humidity assumptions, and construction sequence can change the loss calculation substantially.

There are also limitations that define how the results should be used. The study is an analytical verification of project data rather than an experimental validation. Seismic demand, substructure behavior, foundation response, construction-stage stability outside the modeled checks, cost optimization, and construction scheduling are excluded. The final design assessment is therefore limited to the superstructure and to the load cases explicitly analyzed in this study. For broader design certification, those excluded aspects would need to be addressed within the full project design process. Within that defined scope, the numerical margins are coherent. The deck punching-shear resistance is more than twice the factored wheel-related shear demand, the deck deflection is approximately one fifth of its stated allowable value, the girder reduced flexural resistance exceeds the governing ultimate moment, and the reduced shear resistance exceeds the critical support shear. The final Service I deflection is also substantially below

the adopted limit. These relationships support the conclusion that the selected superstructure dimensions and reinforcement meet the required strength, stiffness, and serviceability criteria for the investigated span.

CONCLUSION

The structural analysis of the STA 109+873 overpass indicates that the analyzed superstructure satisfies the design criteria adopted in this study for the investigated non-seismic loading scope. The 250 mm reinforced-concrete deck slab is adequate with D16-125 main reinforcement at both support and span regions and D13-200 distribution reinforcement. Its factored punching-shear demand of 283.500 kN is lower than the reduced resistance of 761.191 kN, and the calculated 1.958 mm deflection is below the 9.583 mm allowable value.

The diaphragm system also satisfies the local member checks. The 400 x 650 mm support diaphragm and 250 x 1250 mm intermediate diaphragm use 6D19 main reinforcement. The support diaphragm uses 2D13 distribution reinforcement and the intermediate diaphragm uses 4D13; both use D13-200 shear reinforcement. The calculated reinforcement ratios, flexural demands, and shear demands remain within the adopted criteria.

The main 35.80 m PC I-girder H-170 is adequately prestressed with four tendons containing 76 strands. The detailed prestress-loss analysis gives a total loss of 647.743 MPa or 34.825%. After losses, the staged concrete stress checks remain acceptable. The reduced flexural resistance of 21,551.886 kNm exceeds the governing ultimate moment of 15,865.092 kNm, and the reduced shear resistance of 1,617.308 kN exceeds the critical shear demand of 1,270.771 kN. The final Service I deflection of 0.015 m is below the 0.119 m limit used in the analysis.

Future work should extend the analysis to the aspects excluded from this paper, particularly seismic action, construction-stage effects beyond the present calculations, substructure and foundation behavior, and cost evaluation. Independent structural-software verification of the full bridge model would also provide an additional check against the analytical and spreadsheet-based calculations.

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